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Limitations of cyclic pile load tests by kentledge system in soft clay soil

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ABSTRACT

The paper describes the inadequacies of cycled head-down load tests on two barrettes and one bored pile installed in soft clay soil region in Binh Thanh district, and district 7, Ho Chi Minh City, Vietnam, respectively. The soil profile of these sites consisted of layers of organic soft clay and silt from 22.5 m to 28.6 m depth on compact silty sand or semi-stiff to stiff clays to about 60 m depth and followed by dense to very dense sand. The cross-section area of two barrettes located on the Tan Cang complex area was 2,800 mm by 800 mm, which were constructed using the bucket drill technique with bentonite slurry into 65 m depth. The bored pile of the Lakeside project in district 7 having a pile diameter was 1200 mm and 80 m depth. All instrumented piles were attached from ten to eleven strain gages levels along the pile shaft to record the deformation data during the load tests. The strain data analysis shows that the shaft frictions of pile portions located in the soft clay soil regions were increased dramatically, and the base resistances were smaller expected by the setting-up of Kentledge and the cyclic loading tests.

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1 Introduction

The static load tests using the kentledge system have been used widely in Vietnam because this test method can be describing the loads applied on a pile head similar to the weights and live loads of structure during the used period. However, for the bored or barrette piles having large dimensions that are constructed to support the high-rise building in soft soil area, the cycled head-down load tests using the kentledge system become problems for the reliability of recorded strain data and the analysis of test results.

According to Vietnamese National standard TCVN 9393: 2012 (Piles - Standard test method in situ for piles under axial compressive load), before testing, a kentledge system is set up over the pile heads with a total weight of 20% greater than the maximum test load. This load is placed directly on the platform prepared for the tested pile, so its effect may be transferred into the soft soil layers surrounding the pile head. Moreover, the load test in many cycles also causes difficulties for analysis strain data due to the residual load induced after each unloading-reloading phase. The recent studies indicate that the skin friction in the upper pile portion and the elastic modulus of the test piles are overestimated, while the base resistance is

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underrated if the residual stresses in the piles are not considered [1-3]. Besides, the setting-up of the kentledge system directly on the soft soil layers is the main cause of the considerable increase of pile shaft resistances, which are located in these soil strata [4]. Furthermore, the reviews of B.H. Fellenius et al. have shown that an instrumented static loading test, be it a head-down test or a bidirectional test, performed, as it should, in a series of equal load increments, held constant for equal time, and incorporation no unloading-reloading event, will provide suitable data for analysis the load distribution and bearing capacity of the test piles [5, 6].

The paper presents the results of static load tests using the kentledge system on two barrette piles of Tan Cang complex area and one bored pile used in the Lakeside tower, Ho Chi Minh City, Vietnam and discusses the above-mentioned problems.

2 Soil condition

The soil profile of borehole 5^{th} located next to two barrette piles consists of organic very soft clay to about 28.6 m depth on medium stiff clay to 43 *m* depth followed by medium dense to dense sand to 63 *m* and underlain by dense coarse sand to at least 90 m depth [7]. The pore pressure distribution is hydrostatic and corresponds to a groundwater table at 0.8 m depth below the ground surface.

The same as the Tan Cang area, the site profile at Lakeside project consists of an upper layer of organic soft clay soil, which is highly compressible and normally consolidated. The soil profile consists of grey-green very soft to soft clay to about 22.5 *m* depth, followed by alternating layers of medium stiff clay, clay sand, and dense sand to 72.7 *m* depth underlain by hard clay to at least 90 *m* depth [8]. Table 1 summarizes the soil parameters of the soil layers at two sites.

Table 1 – Soil Parameters

Items	Unit	Tan Cang complex area				Lakeside tower				
		Layer I	Layer II	Layer III	Layer IV	Layer 1	Layer 2	Layer 3	Layer 4	Layer 5
Saturated density (ρ_{sat})	kg/m^3	1470	1850	1910	2020	1493	1962	1710	2022	2018
Void ratio (e_0)	-	2.48	0.82	0.61	0.54	2.16	0.67	0.57	0.52	0.54
Water content (w_n)	%	88.7	31.7	28.5	17.4	81.9	26.3	18.3	15.1	16.1
Liquid limit (LL)	%	80.6	40.5	-	-	66.6	34.8	22.6	-	30.2
Plastic limit (PL)	%	31.5	20.8	-	-	30.9	17.2	6.2	-	12.3
Friction angle (Φ)	(^{0})	$4^0$08’	$9^0$38’	$36^0$06’	$42^0$08’	$3^0$21’	$10^0$45’	$29^0$45’	$34^0$04’	$18^0$09’
Cohesion (c)	*kPa*	7.3	15.2	5.1	2.1	6.4	15.1	10.2	8.7	63.2
SPT index range	*bl*/0.3	0	11-35	17-41	>50	0	14-26	16-24	18-50	30-38

3 Details of the instrumented piles

Two test piles (namely BR1 and BR2) in the Tan Cang complex area had a rectangular cross-section area of 2,800 *mm* by 800 *mm* and were supplied with a reinforcing cage of forty 25-*mm* bars, resulting in a steel reinforcement area of 196.3 cm^2 and a reinforcement ratio of 0.88 % for the 2.24 m^2 nominal pile cross-section. These barrettes were constructed to 65 *m* depth, using grab-bucket excavation techniques with bentonite slurry. Fig.1a indicates the locations of the strain-gages levels attached along with two barrette piles. Each gage level (GL) contained four strain gauges are arranged at the 4 corners of a barrette cross-section.

Shaft grouting was carried out on the barrette BR1 after completion of concrete placement over about 7.4 *m* above toe level. To evaluate the efficiency of the grouting technique, Pile BR1 was attached eleven gage levels, which has one more level than that of Pile BR2.

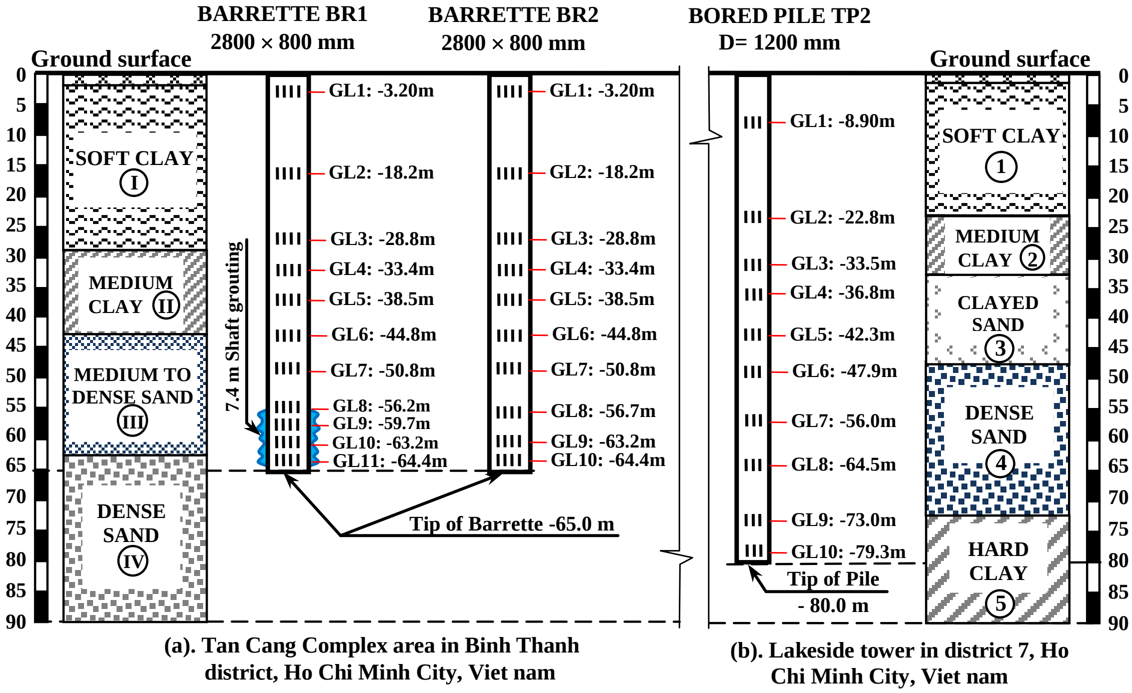

Figure 1 – Details of the instrumented piles

To verify the capacity of the designed piles for the 23-story buildings of the Lakeside project, the bored pile, in 1,200 mm diameter, was installed into 80 *m* depth on December 04, 2016, to serve the head-down tests using the kentledge system. This pile was supplied with a reinforcing cage of sixteen 25 *mm* bars (Fig.2a) and instrumented with three vibrating wire strain-gauges on each cross-section area at ten levels below the ground surface as shown in Fig.1b.

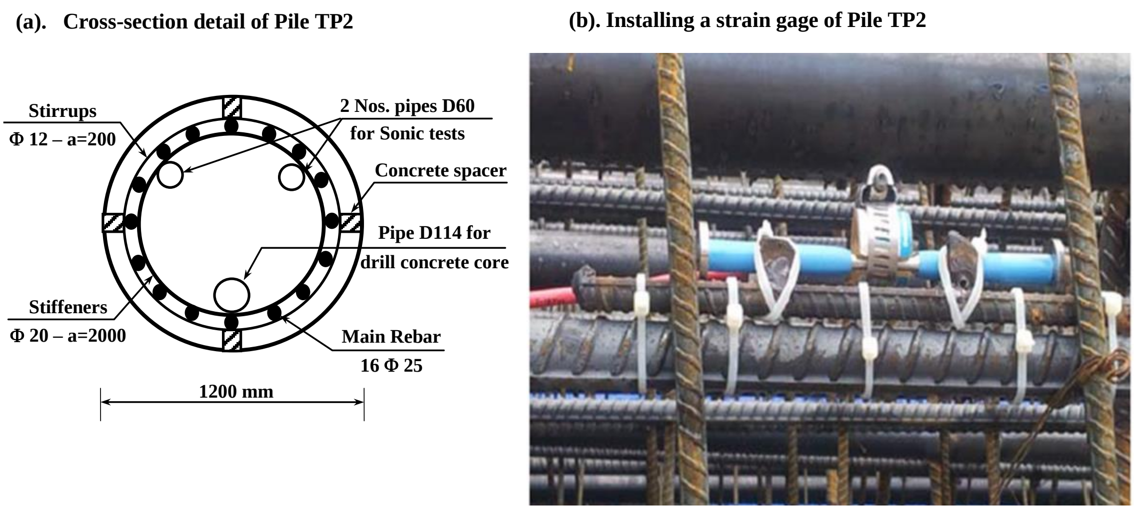

Figure 2 – Details of Pile TP2

4 Load tests and measurements

4.1 Loading schedule

The static load test for Pile BR1 was carried out in two phases [9]. Phase 1 loading was performed by four load increments to an intended load of 1800 T and unloading in four steps. In phase 2, this pile was first reloaded to the 1800 T loads in two increments, and then the loading continued to reach the maximum load of 4500 T. However, Pile BR2 was loaded test in three loading cycles to the maximum loads of 1300 T, 3250 T, and 4500 T, respectively. Similarly, the testing of Pile TP2 was carried out in three phases to the intended loads of 1100 T, 2200 T, and 2750 T, respectively [10]. Almost all load levels were maintained for 60 minutes, except for the last levels of each cycle were held for 24 hours.

4.2 Results of the load tests

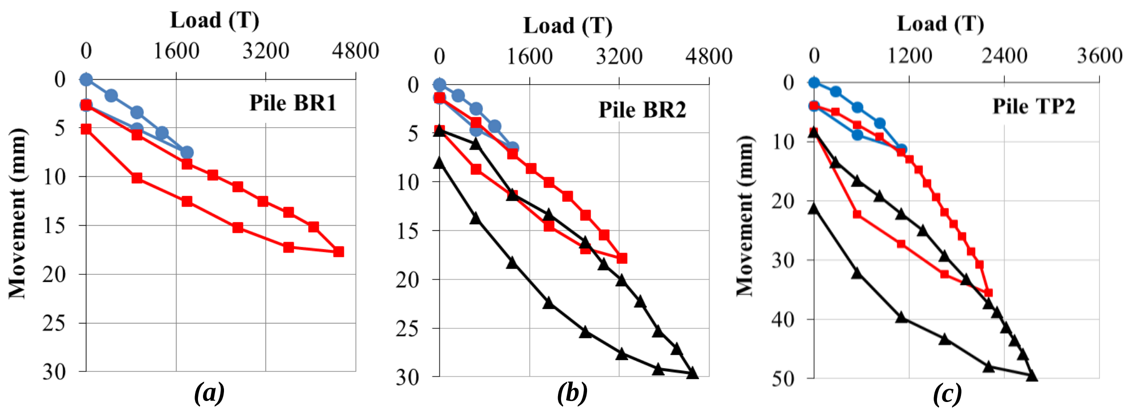

Fig.3 - Load – movement curves of three tested piles

Fig.3 represents the relationship of loads and the pile head movements of three tested piles. The maximum movements of Piles BR1 and BR2 were about 17.7 *mm* and 29.6 *mm*, respectively, measured at the same applying load of 4500 T. Besides, the maximum test load and measured settlement of Pile TP2 were about 2750 T and 49.5 *mm*, respectively.

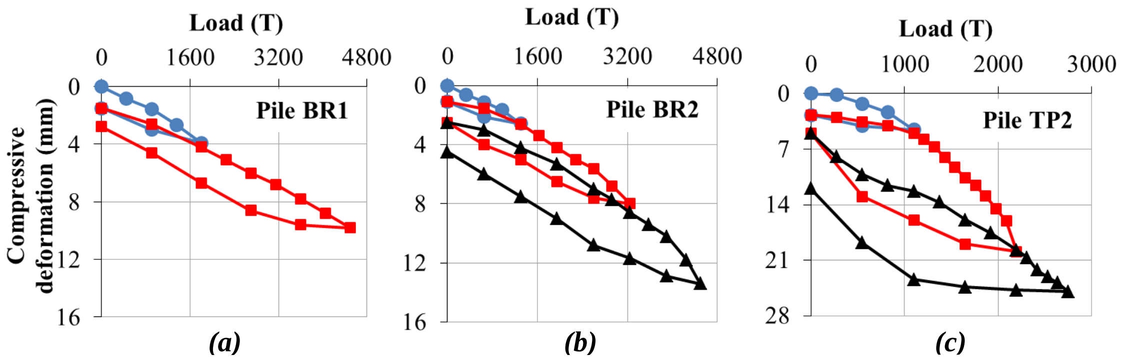

Fig.4 – Load – compressive deformation curves of three tested piles

Generally, the movement of a pile head consists of three components: the compressive deformation of the pile, the settlement caused by the load transmitted along the pile shaft, and the settlement due to the load at the tip. Barrette BR1 is the shaft-grouted pile and tested only 2 cycles. Thus, the shortening of this pile at the same applying load is clearly small than that of Pile BR2, as indicated in Figs. 4a and 4b. Furthermore, the graphs in Figs. 4b and 4c show that the residual deformations of the Piles BR2 and TP2 caused by unloading-reloading events are significant; therefore, they affect noticeably the shapes of the corresponding load-movement curves in Cycle 3. It can be observed that the largest shrinkage of Pile TP2 was 25 mm caused by the pile head load of 2750 T; consequently, the stiffness value of this pile is smaller noticeably than those of Piles BR1 and BR2.

4.3 Modulus of the tested piles

Normally, to convert the measured strain to the load values, it is necessary to know the modulus (E) of the pile material. C. Lam et al. and B.H. Fellenius indicated that one of the best ways of estimating the pile modulus is using a so-called "tangent modulus" or "incremental stiffness" plot, that is, the applied increment of the load over the induced increment of strain plotted versus the measured strain [11-13]. The relationship between the vertical compressive stiffness (EA) and strains are shown in Fig.5 for all studied piles. Note that, only the records of the first strain gages (GL1) were used in determining the stiffness magnitudes. These curves of Piles BR1, BR2, and TP2 suggested the stiffness values of these piles are 96 GN; 88 GN and 27.5 GN, respectively.

Usually, the modulus of a concrete pile reduces with increasing stress or strain. This means that the relationship between the applying load and the compressive deformation of a pile follows a curve, not a straight line. However, the load-compress graph at the loading phase in Cycle 2 of Pile BR1 is nearly linear, so the value of stiffness (EA) does not change insignificantly, as shown in Fig. 5a. In contrast, over the large strain range imposed during the load tests of Piles BR2 and TP2, the difference between the initial and the final modulus for these concrete piles can be considerable.

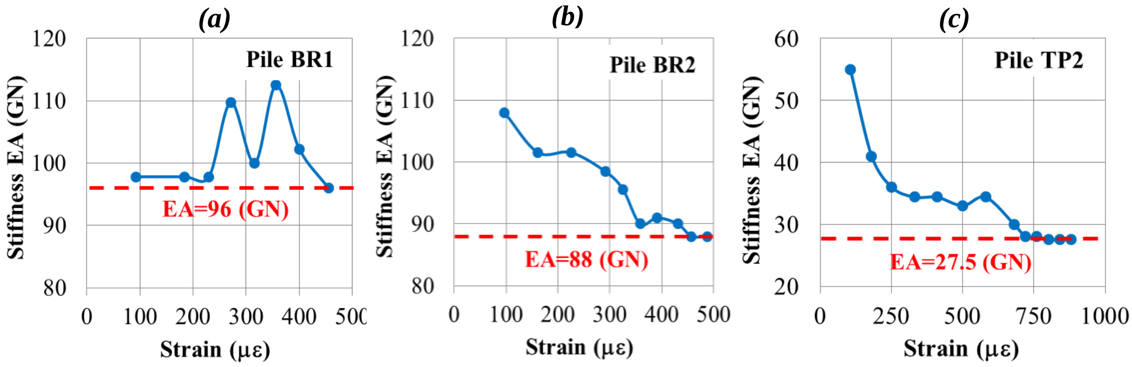

Figure 5 – Stiffness plot of records from gage level 1 (GL1) of three tested piles

4.4 Load distribution along the pile shaft

Using the axial stiffness (EA) to convert the measured strain to load (Q_i) by the equation:

$$Q_i = EA\varepsilon_i \tag{1}$$

Where: Q_i is the load distribution at the level "*i*", EA is the vertical compressive stiffness of a pile, and ε_i is the average measured strain at level "*i*".

The counterweight of the kentledge systems used for the load tests of Piles BR1 and BR2 were about 5500 T. Each support of this system carried on about 2750 T and this load was distributed on an area of about 55 m^2. As a result, the counterweight was a preload on the soft clay layer around the test piles and this preload improved markedly the shear resistance of the first soil layer.

As can be seen clearly in Fig. 6, at the maximum load of 4500 T, the shaft friction of Pile BR1 measured from ground surface to about 28.8 *m* depth (GL3 level) had become significant to reach the value of 1330 T, which accounted for about 29.6% of the pile head load. Therefore, the average unit shaft resistance of this pile segment is 88.8 *kPa*. However, the maximum value of base resistance conducted at the end of Cycle 2 was only 259.2 T, which was approximately about 5.7% of the maximum applying load. These values are unexpected and do not accurately reflect the actual shear resistance of the soft clay soil located in the surrounding pile-top and the dense sand layer at the pile toe. Moreover, the comparisons of load distribution along pile shaft of Pile BR1 conducted at the load levels of 900 T and 1800 T clearly show that the load sharing at the same gage levels in Cycle 1 is totally smaller than corresponding results in Cycle 2, because of the residual load caused by the unloading-reloading process.

Similarly, the load-transfer curves of Pile BR2 shown in Fig.7 plainly inform that at the maximum load of 4500 T, the shaft resistance of the pile segment located in the soft clay layer was 900 T, therefore, the corresponding value of unit skin friction was 43.4 *kPa*. However, the biggest value of base resistance was only 158.4 T, which accounted for approximately 3.5% of pile capacity. Note that at the first 3 levels of Cycle 3, the residual loads caused the axial forces at the gage GL1 of Pile BR2 were clearly larger than the corresponding applying loads.

Besides, it is surprised to see that at the same pile head load of 4500 T, the maximum base resistance of Pile BR1 (259.2 T) considerably greater than that of Pile BR2 (158.4 T), although the former pile was used the shaft-grouting technique and the remaining is a plain pile. It strongly shows that if using more cycles in the static load tests, it made to underestimate the toe resistances.

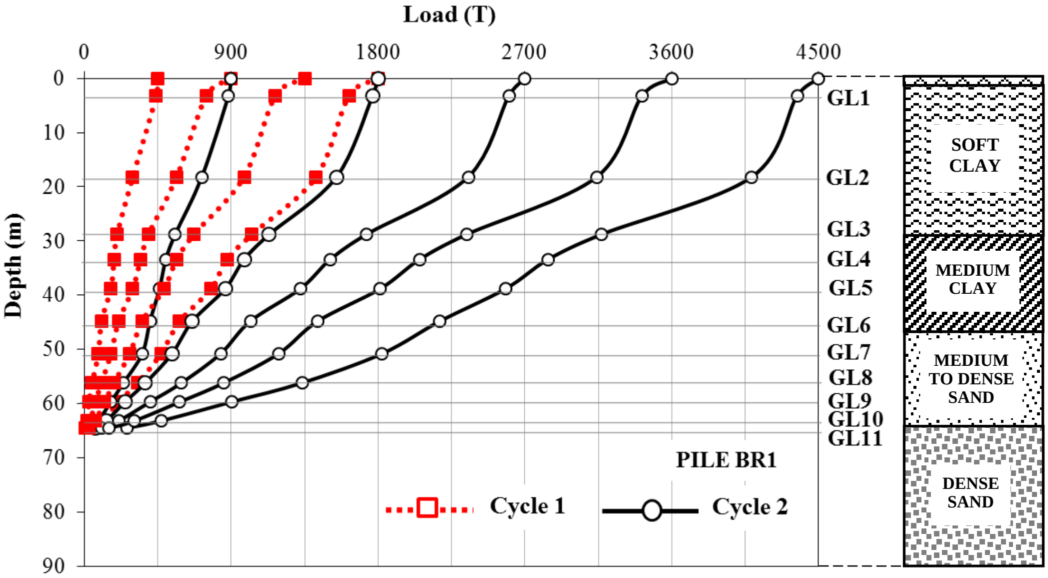

Figure 6 – Load distribution of Pile BR1

Turning to Pile TP2, the largest pile head load and the shaft resistance from the pile top to the GL2 level were about 2750 T and 386 T, respectively. Thus, the average unit shaft friction of this pile portion equals about 44.9 *kPa*. This value is markedly greater than expected, because the SPT N-indices of this soil layer is zero, as shown in Tab.1. In contrast, the maximum value of the pile base load was only 382.2 T, and accounted for 13.9% of the working load, despite the fact that the pile toe was located in the hard clay layer. Because of the presence of residual load, the measured loads indicated, falsely, that the soil resistance acting against the pile tip was very small. Also, we can see the irrationality of some graphs in the initial loadings of Cycle 3, because the values of the load distribution at GL1 levels of Pile TP2 are markedly larger than the corresponding loads imposed at the pile head as indicated in Fig.8.

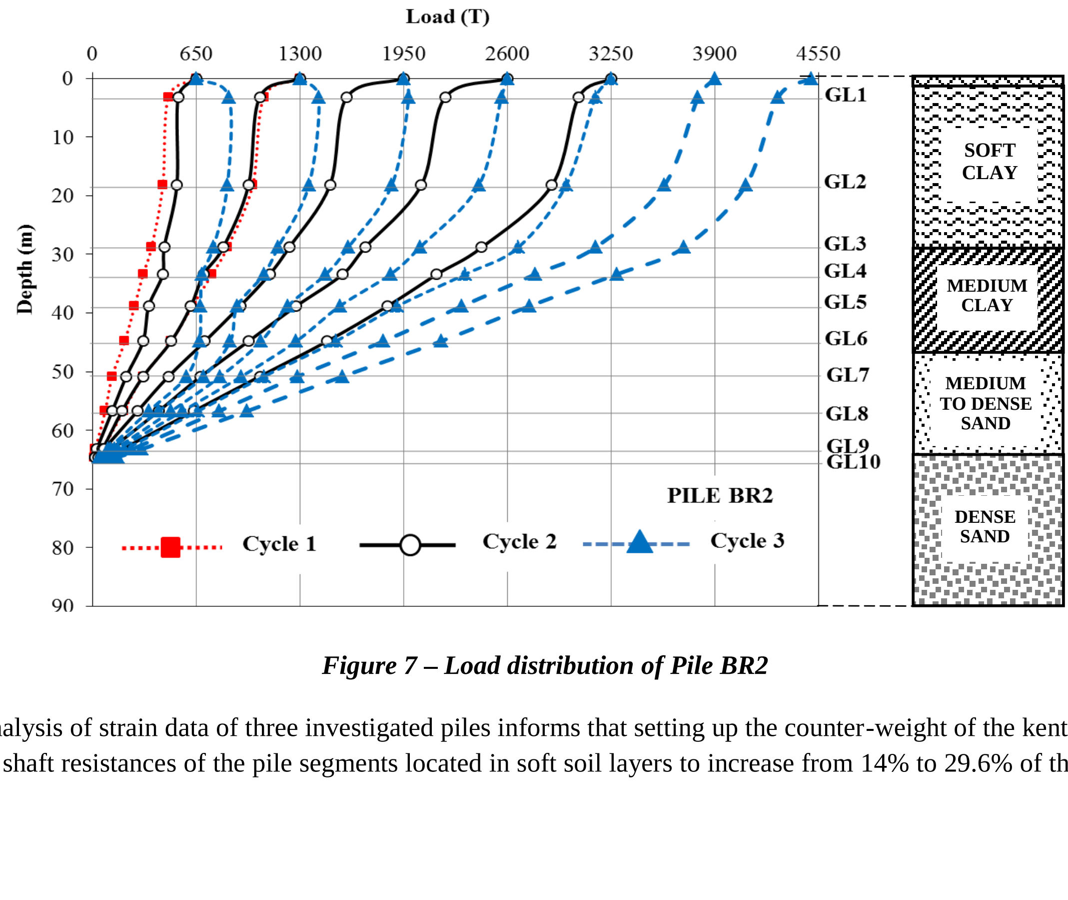

Figure 7 – Load distribution of Pile BR2

The analysis of strain data of three investigated piles informs that setting up the counter-weight of the kentledge system caused the shaft resistances of the pile segments located in soft soil layers to increase from 14% to 29.6% of the maximum-loads applied on the corresponding pile heads. This leads to the shaft resistance at the lower depths and the capacity of and pile base not mobilized fully, so the working loads of the tested piles have not obtained the expected results.

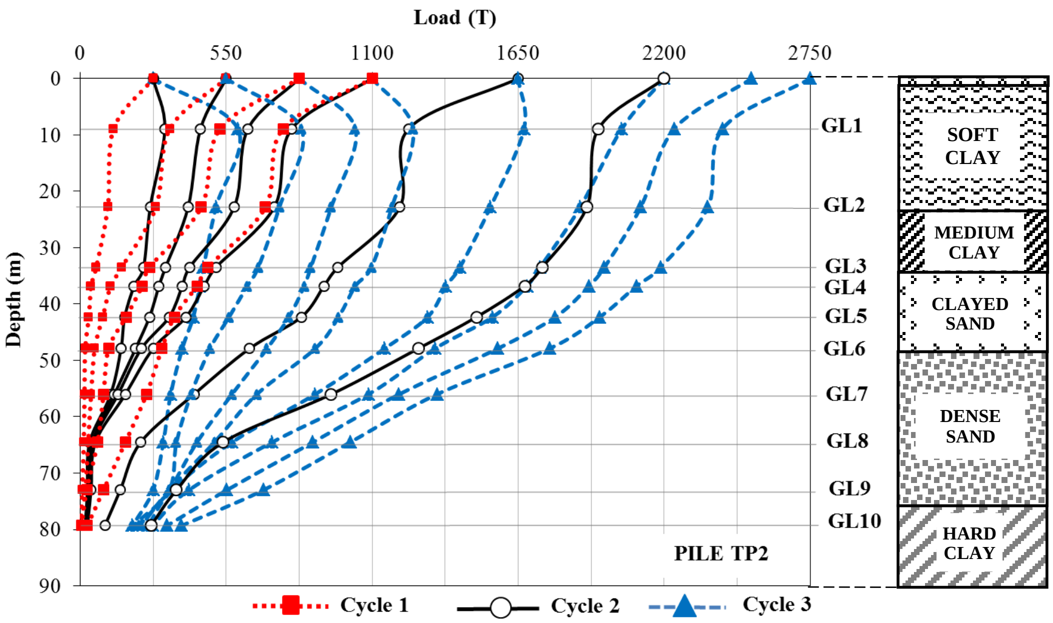

Figure 8 – Load distribution of Pile TP2

On the other hand, the series of load distribution curves of the tested piles clearly show that the magnitudes of the load distribution at the same level conducted by the equal applied load increase significantly by the loading cycles. It provides more evidence to inform that, using more unloading-reloading phases in the load test processes, consequently, get more the residual loads presented in these piles. Moreover, these residual loads of Piles BR2 and TP2 were accumulated for the reloading of Cycle 3 and it leads to the unreasonable load distribution curves as indicated in Figs 7 and 8. The cycled head-down load tests on two piles that have developed residual load are subjected to negative skin friction along the upper length of these piles, which are resisted by positive shaft frictions in the lower part of the piles and the corresponding toe resistances. So the presence of residual loads will result in an underestimation of toe resistances.

5 Conclusion

The static load tests and the measurements of Piles BR1, BR2, and TP2 constructed in soft clay soil regions were presented. The analysis of the test results was pointed out the problem of cyclic pile load tests by the kentledge system. The following conclusions can be drawn from the present study.

The average unit frictions in the soft clay layer obtained by strain data of three instrumented piles range of 43.4kPa through 88.8 *kPa*. These values are larger significantly than required and they indicate that the counterweight of kentledge systems cause a significant change of the mechanical properties of the top soft soil layers.

The measured load distributions are not the true load sharing along the piles, as the piles are subjected to residual load; therefore the toe resistances of barrette Piles BR1 and BR2 were only 259.2 T and 158.4 T, respectively measured at the pile head loads of 4500 T. Similarly, the pile tip load of Pile TP2 accounted maximum of about 13.9% of the pile capacity. These soil resistance of the pile tips were smaller expected and did not reflect the actual soil conditions at the sites, because the SPT indices (N) at the corresponding depths were all larger than 30.

The load distribution curves of the tested piles BR1, BR2, and TP2 indicated that the static load test in many loading cycles affected significantly the strain measurements and added difficulties for estimating the shaft resistances and corresponding underestimation of the toe resistances. Depending on the magnitude of the residual loads, the measured shaft and toe response can vary considerably.

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